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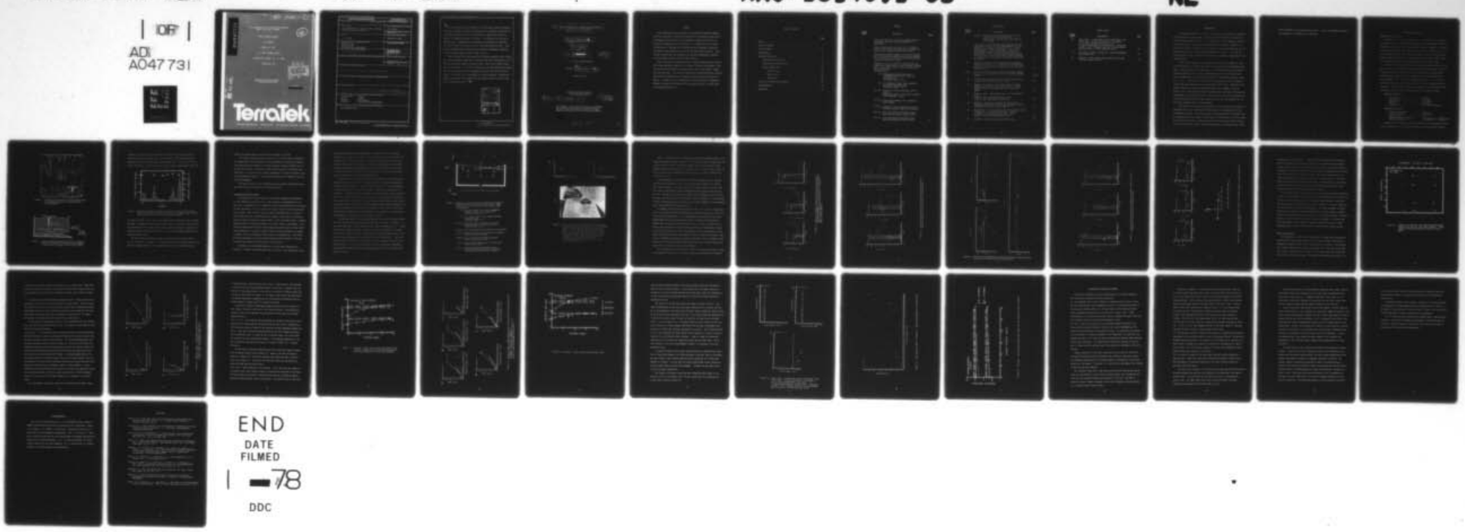

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FIELD EXPERIMENTS ON THE CASTLEGATE SANDSTONE,
RANGELY ANTICLINE, COLORADO

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FINAL TECHNICAL REPORT

H. S. SWOLFS

OCTOBER 31, 1977

U. S. ARMY RESEARCH OFFICE

CONTRACT NO. DAAG29 - 76 - C - 0003

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Field experiments were performed in the Castlegate Sandstone (Rangely Anticline, Colorado) to measure the mechanical and transport properties of a rectangular block (2 m^3) of rock containing a gouge-filled joint as a function of compressive stress to 30 bars. In preparation of the static loading tests, a thorough site investigation was conducted to determine the ambient stress and velocity field in the undisturbed rock mass. These studies revealed that the fractured, but otherwise homogeneous sandstone is pre-stressed to about 10 bars (1 MPa; 145 psi).

Static loading tests performed on the isolated sandstone block indicate that either permanent compaction, or dilation and shear displacement of the calcite-filled joint begins at applied stress levels only slightly higher than the pre-stress. The results of these tests provide additional evidence that joint displacement can be triggered by distant high-yield sources of energy. The impact of this phenomenon on siting technology is emphasized. Attempts to measure fluid permeability with applied stress in these tight joints proved unsuccessful.

6 FIELD EXPERIMENTS ON THE CASTLEGATE SANDSTONE, RANGELY ANTICLINE, COLORADO

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SUMMARY

Field experiments were performed in the Castlegate Sandstone (Rangely Anticline, Colorado) to measure the mechanical and transport properties of a rectangular block (2 m^3) of rock containing a gouge-filled joint as a function of compressive stress to 30 bars. In preparation of the static loading tests, a thorough site investigation was conducted to determine the ambient stress and velocity field in the undisturbed rock mass. These studies revealed that the fractured, but otherwise homogeneous sandstone is pre-stressed to about 10 bars (1 MPa; 145 psi).

Static loading tests performed on the isolated sandstone block indicate that either permanent compaction, or dilation and shear displacement of the calcite-filled joint begins at applied stress levels only slightly higher than the pre-stress. The results of these tests provide additional evidence that joint displacement can be triggered by distant high-yield sources of energy. The impact of this phenomenon on siting technology is emphasized. Attempts to measure fluid permeability with applied stress in these tight joints proved unsuccessful.

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INTRODUCTION

The mechanical behavior of rock in its natural state, when it is subjected to quasi-static or dynamic loads, is not so much controlled by the portions of solid rock but, rather, by the *defects* in the rock - joints, fractures, bedding planes, and faults - and the infill material they may contain. Because it is difficult to fully characterize rock mass behavior in the laboratory due to size and load limitations, a strong case can be made for field (*in situ*) tests of various kinds on large, representative samples to complement a geotechnical site investigation.

In this final report, we will summarize the results of a number of field tests performed on a jointed (fractured) 2 cubic-meter block of Castlegate sandstone. This series of field experiments is a continuation of a basic research program designed to obtain a better understanding of the role of joints and other discontinuities on the mechanical behavior of fractured rock masses under *static loads*. The results of an earlier field program on a block of Sherman granite have been documented by Pratt and others (1974, 1977). Research on the present contract was conducted during the period from September, 1975, to August, 1977, at the Rangely Anticline, Colorado, test site and at the Terra Tek laboratories in Salt Lake City, Utah. This particular test site in Colorado was chosen because the behavior of the jointed sandstone was anticipated to be significantly different from a jointed granite.

The report will begin with a description of the regional and site-specific characteristics of the Castlegate Sandstone exposed in the Rangely Anticline, northwestern Colorado. Of importance here is the documentation of selected rock-mass properties and the changes incurred during the excavation of the test sample. The results of the static loading tests will be highlighted by detailed descriptions of the uniaxial, biaxial and shear experiments and contrasted with similar

tests performed on the Sherman granite block. Finally, the potential impact of the research on technology will be outlined.

TECHNICAL DISCUSSION

Geology of the Test Site

The test area (T. 2 N.; R. 103 W.; Sec. 4; see Figure 1) is located on the northern flank of the Rangely Anticline, site of the largest oilfield in northwestern Colorado. This west-north-west trending structure is a simple, elongate, plunging monoclinal flexure with surface dips varying from 2^o - 4^o on the northern flank to $15^0 - 40^0$ on the southern flank. In this area, the exposed Castlegate Sandstone is the lowest member of the Price River Formation (Mesaverde Group) and overlies the Mancos Shale (Hale, 1959).

Cullins (1969) divides the Castlegate Sandstone into two units; (1) the upper unit is a very light-gray to gray (weathers same), massive, limy and fine-grained lagoonal deposit, and (2) the lower unit is a gray-orange, yellowish-brown weathering, limy, friable and very fine-grained littoral marine sediment. The test block was excavated in the *lower unit* of the Castlegate Sandstone about 9 meters north of a shallow, south-facing cliff (Figure 2). The physical and mechanical properties of this sandstone unit, measured on selected cores in the laboratory, are listed as follows:

Physical Properties

Grain Size	0.05 mm
Grain Density	2.63 gm/cc
Bulk Density	1.97 gm/cc
Porosity	25 percent
Permeability	190 ± 80 millidarcies

Mechanical Properties

Compressive Strength	114 bars	
Young's Modulus	23 kilobars	
Poisson's Ratio	0.28	
Compressional Wave Velocity	2.1 km/sec (dry)	1.7 km/sec (moist)
Shear Wave Velocity	1.4 km/sec (dry)	1.1 km/sec (moist)

Additional field investigations were conducted in the area near the test site to determine (1) the direction and spacing of near-vertical joints or

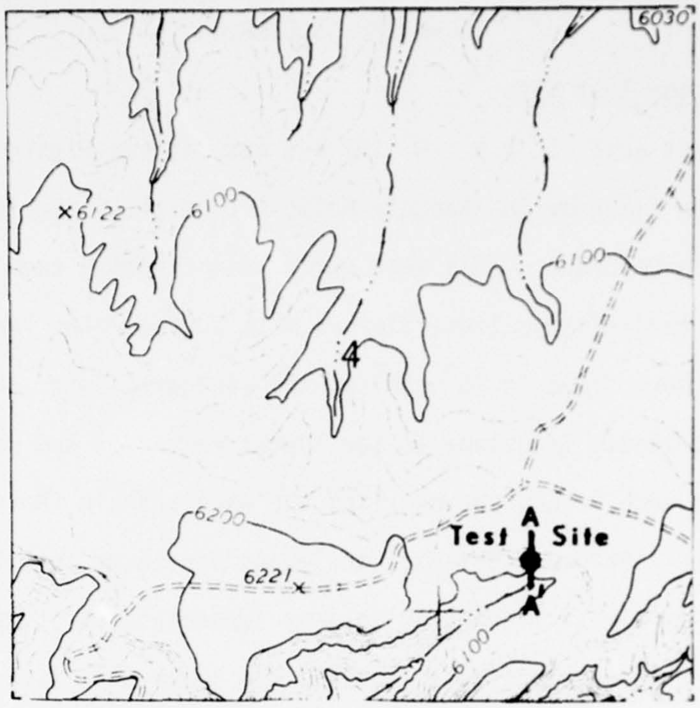


Figure 1. Location of the test site on the northern flank of the Rangely Anticline, Colorado (Mellen Hill Quadrangle)

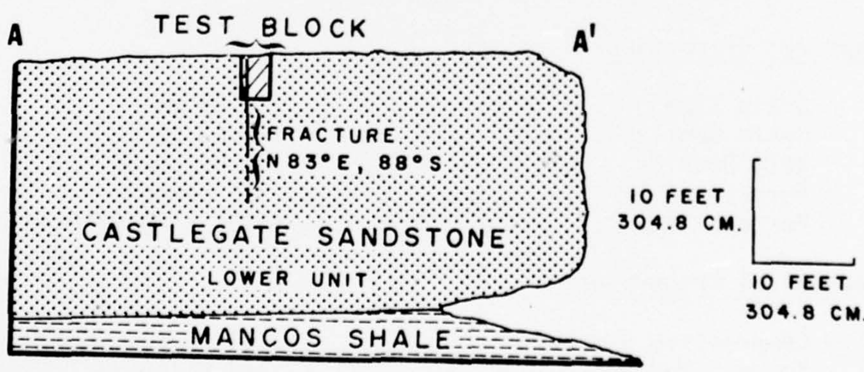


Figure 2. Cross section through test site (A - A', Figure 1) showing lithologies, test block and orientation of near-vertical, gouge-filled fracture (joint)

fractures, (2) the seismic and ultrasonic velocities, and (3) the state of stress in the lower sandstone unit. The fractures in the otherwise massive sandstone unit can be divided into three dominant sets: N 35° W, N 55° E and N 85° E (Figure 3). The first two sets contain rather inconspicuous, hair-line

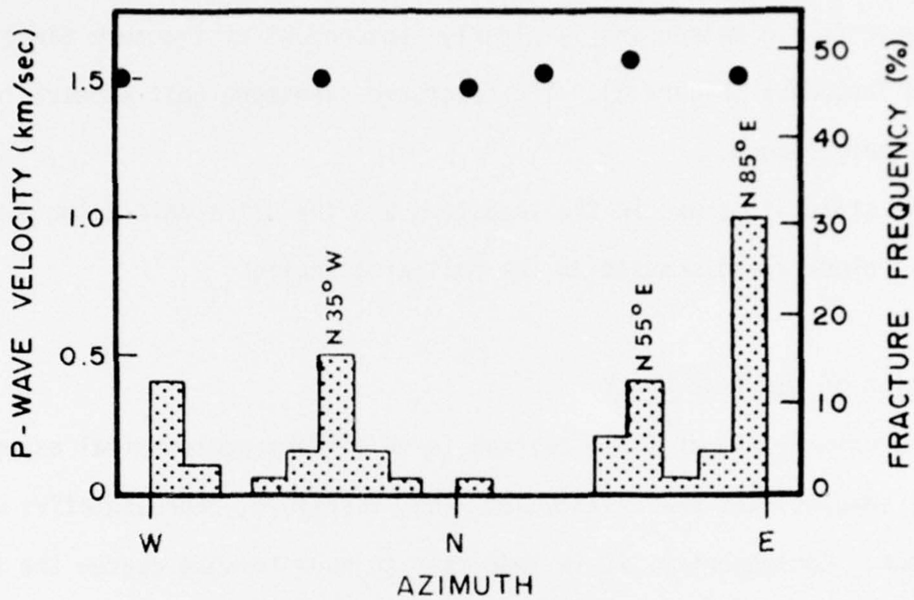


Figure 3. Relationship between frequency of vertical fracture sets (65 fractures measured) and azimuthal distribution of compressional (P) wave velocity measured by the seismic-refraction method.

fractures spaced 60 - 90 cm apart. The third and most conspicuous set, striking east-west, is composed of long, gouge-filled (up to 0.3 cm thick) fractures that are spaced about 180 cm apart; their spacing, however, decreases to about 30 cm toward the edge of the south-facing cliff. This set of fractures appears to be a stress-relief feature.

The test block was located adjacent to one of these joints that strike N 83° E and dips 88° S (Figure 2). The width of the joint averages about 0.15 cm. The infill (gouge) material is composed almost entirely of compacted calcite

grains; the average grain size of the calcite gouge is 0.02 mm.

Five seismic-refraction profiles were shot in the test area to determine the compressional (P) wave velocity in the sandstone as a function of azimuth. True velocities were obtained over travel distances of about 23 meters by reversing the profiles. The average seismic P-wave velocity in the lower sandstone unit is 1.5 km/sec and is clearly independent of fracture direction, spacing and frequency (Figure 3). The fractured sandstone unit appears to be seismically homogeneous.

The state of stress in the sandstone and the ultrasonic velocities across the test block are discussed in the following section.

Excavation of the Test Block

The primary aim of field testing is to perform geotechnical experiments on rock samples that are *undisturbed* and, therefore, representative of the host rock. Consequently, it is important to know to what degree the rock sample has been disturbed, if at all, during its required isolation from the surrounding rock mass. The principal causes of sample disturbance are: (1) change in stress conditions due to unloading and (2) change in rock fabric or structure resulting from the excavation process itself. While the latter effect is usually negligibly small in large rock samples, the former effect is unavoidable. The material changes that take place during sample disturbance due to stress relief can be measured by a variety of methods; these include measurements of change in stress, strain, displacement, sonic velocity and permeability. The results of these measurements will be presented following the description of the test block and the procedures followed during its excavation.

A suitable site was selected adjacent to an east-west trending joint (Figure 2). Within a two square-meter area, but prior to the excavation of the

block, surface locations were prepared for the application of strain-gage rosettes (S_1, S_2 and S_3; see Figure 4), and shallow holes were drilled for the emplacement of a stress gage (SG), a displacement gage (DG) and a ultrasonic velocity transmitter (VT) and receiver (VR). At some distance from the block location an additional strain-gage station (S_4) was prepared to be overcored at some later time and a vertical hole (P_4) was drilled to allow the measurement of permeability in the undisturbed sandstone.

The excavation of the block itself was accomplished by line-drilling and broaching three vertical slots 1.2 meters deep and 3 cm wide using pneumatic equipment (Figures 2 and 4). The two side slots were 90 cm long and extended 15 cm beyond the joint. The third slot was 2.1 meters long and parallel to the joint. The resulting block, bounded on the north side by the near-vertical joint, was essentially free along its sides but remained attached to the host rock at the bottom. This particular slot pattern was determined by the results of computer-model studies using the finite-element method (FEM) conducted in advance to ascertain the most appropriate geometry of the test block that would insure uniform loading of the isolated joint. The excavation of the block was accomplished in five days (July 28 to August 1, 1976).

The measured changes in stress, displacement, strain and ultrasonic velocities that resulted from the excavation of the block are shown in Figures 5 - 11. The stress gage (SG), a direct-reading stress meter developed by Dr. H. S. Swolfs at Terra Tek, registered changes (Figure 5) in only two directions: 2.5 bars (N 7° W) and 4.9 bars (N 53° E). Multiplied by a calibrated gage factor of 2.2, the true stress changes were 5.5 bars and 10.8 bars, respectively. The third direction (N 67° W) could not be measured at the time because of lack of transducing equipment. Insufficient information from this particular measurement prevented the calculation of the magnitude and direction of the principal components of stress.

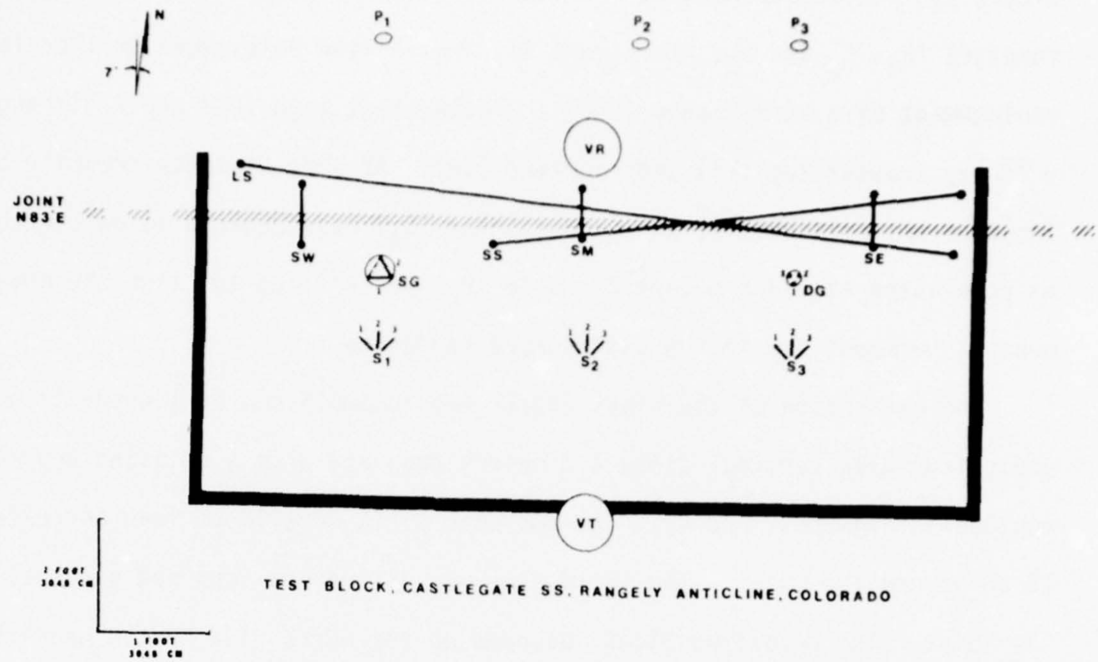


Figure 4. Schematic of the top surface of the test block excavated to include portion of the east-west trending joint. Heavy black lines indicate the position of the vertical slots. Instrumentation:

SG - Three-axes borehole stress gage (STRABOMETER developed at Terra Tek - see Figure 5C). Emplacement depth - 60 cm.

DG - U. S. Bureau of Mines three-component borehole displacement gage. Emplacement depth - 60 cm.

VT & VR - Ultrasonic velocity transducer and receiver. Emplacement depth - from surface to 60 cm. Travel distance - 86 cm.

S_1,S_2,S_3- Strain-gage rosettes (45^o) cemented to block surface.

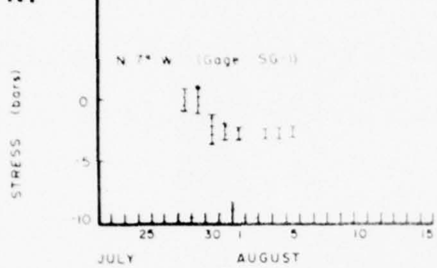
P_1,P_2,P_3- Permeability holes dipping 45^o south to intersect vertical joint at 60 cm depth.

SW,SM,SE- Short gage-length displacement transducers across surface trace of joint.

SS & LS - Long gage-length displacement ransducers along surface trace of joint

S_4 & P_4 - Strain-gage rosette and permeability hole located 3 meters east and north, respectively from the block (not shown on this figure).

A.



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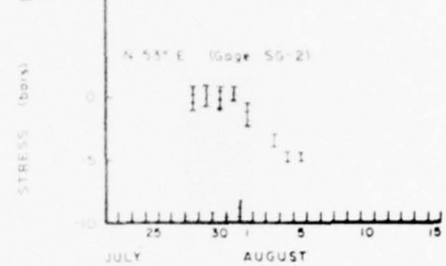

C.

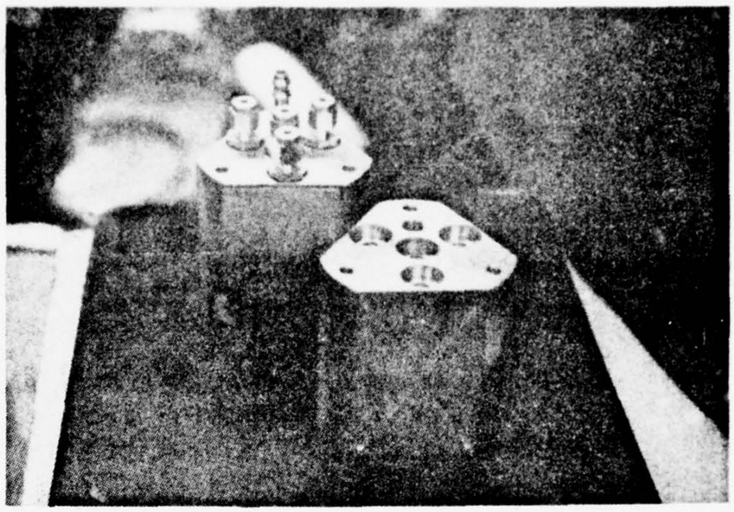

Figure 5. A & B. Stress relief (decompression) monitored on two components of the stress gage (STRABOMETER) at 60 cm depth during excavation of the test block - July 28 through August 1, 1976 (shaded area). Third component inoperative due to lack of instrumentation. Vertical bars in this and subsequent figures indicate the maximum daily variation in the parameter measured.
C. Photograph of the aluminum stress meter used in the block test.

The U. S. Bureau of Mines displacement gage (DG) registered changes in all three horizontal directions (Figure 6). The associated stress changes were calculated using 23 kilobars for Young's Modulus and 0.28 for Poisson's Ratio (Ageton, 1967): 5.5 bars (N 7° W), 10.6 bars (N 53° E) and 19.1 bars (N 67° W). The principal stress magnitudes and directions were 15.7 bars (N 79° W) and 7.7 bars (N 11° E).

Strain changes were monitored by the surface rosettes (S_1, S_2 and S_3), but they show considerable scatter and are unreliable (Figures 7, 8 and 9). Their ineffectiveness is primarily due to the high porosity of the host rock which made it difficult to properly cement the strain gages to the surface of the rock. Rosette S_4, located some 3 meters east of the block, was overcored on August 13, 1976, using a 15-cm diameter core bit (Figure 10). The principal stresses, calculated using 23 kilobars for the Young's Modulus of the sandstone, were 9 bars (N 88° W) and 4 bars (N 2° E).

In summary, the principal orientation of the *in situ* state of compressive stress is east-west and the magnitudes show a consistent two-to-one ratio irrespective of the method used. It is interesting to note that previous stress measurements by de la Cruz and Raleigh (1972) at various sites around the Rangely Anticline have yielded comparable results. Using a variety of techniques, they estimated the surface stress magnitudes to average about 10 bars. The principal directions of stress determined by surface measurements, earthquake focal-plane solutions and hydraulic fracturing (Raleigh and others, 1972) are also reasonably consistent: the maximum horizontal stress component strikes about west-north-west and sub-parallel to the fold axis, and the minimum horizontal stress component strikes north-north-east.

The consequences of relieving about 10 bars of compressive locked-in stress on the mechanical properties of the rock sample were determined by ultrasonic

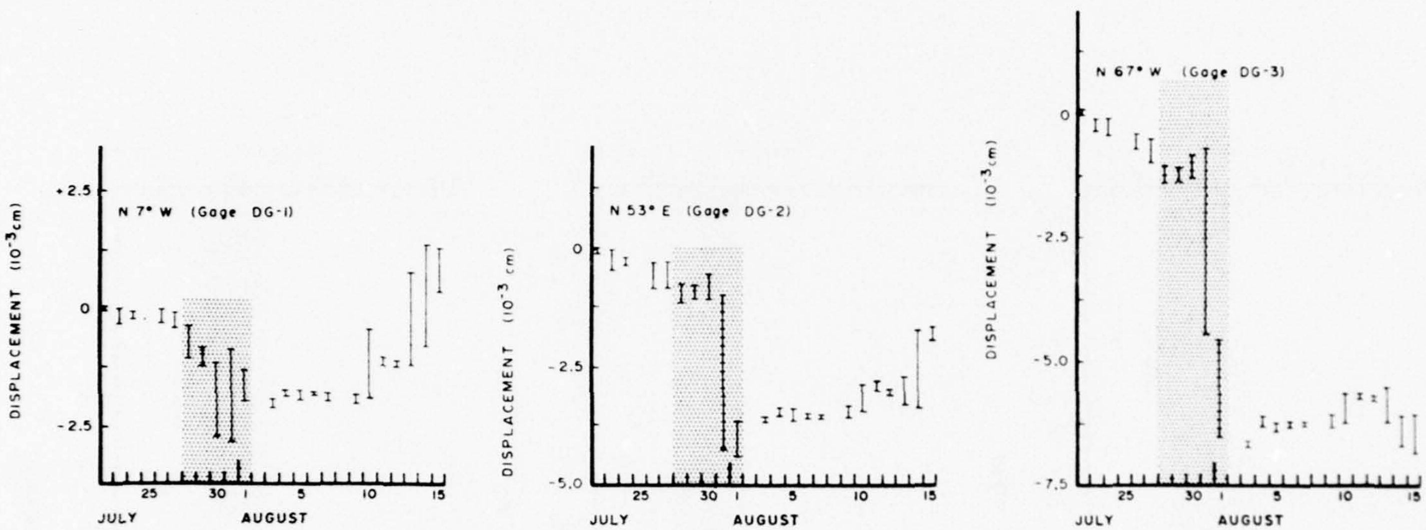

Figure 6. Change in borehole diameter (expansion) monitored by the three-component U.S. Bureau of Mines displacement gage at 60 cm depth during excavation of the test black (shaded region).

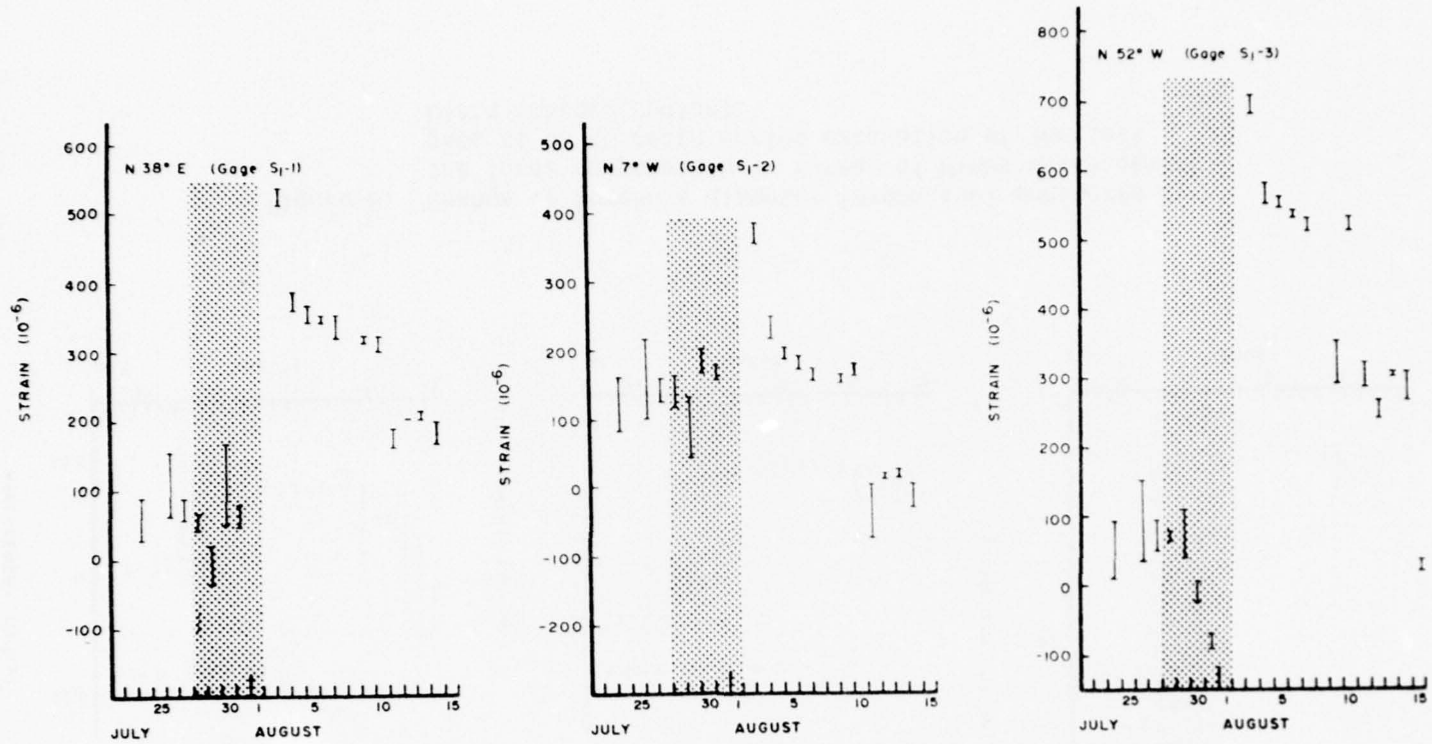

Figure 7 Strain relief monitored by surface strain-gage rosette S_1 during excavation of the test block (shaded region) . . .

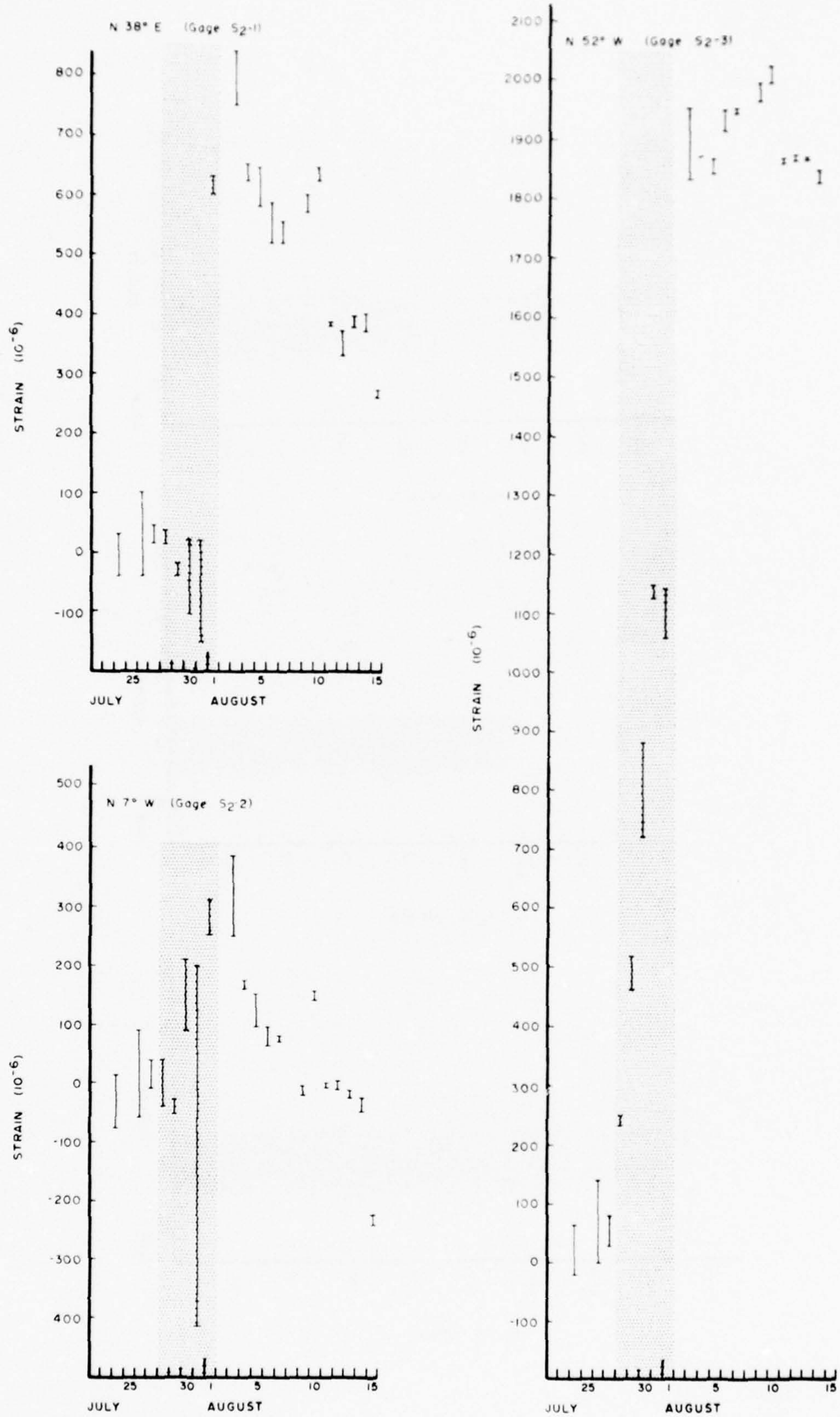

Figure 8. Strain relief monitored by surface strain-gage rosette S_2 during *excavation of the test block (shaded region).*

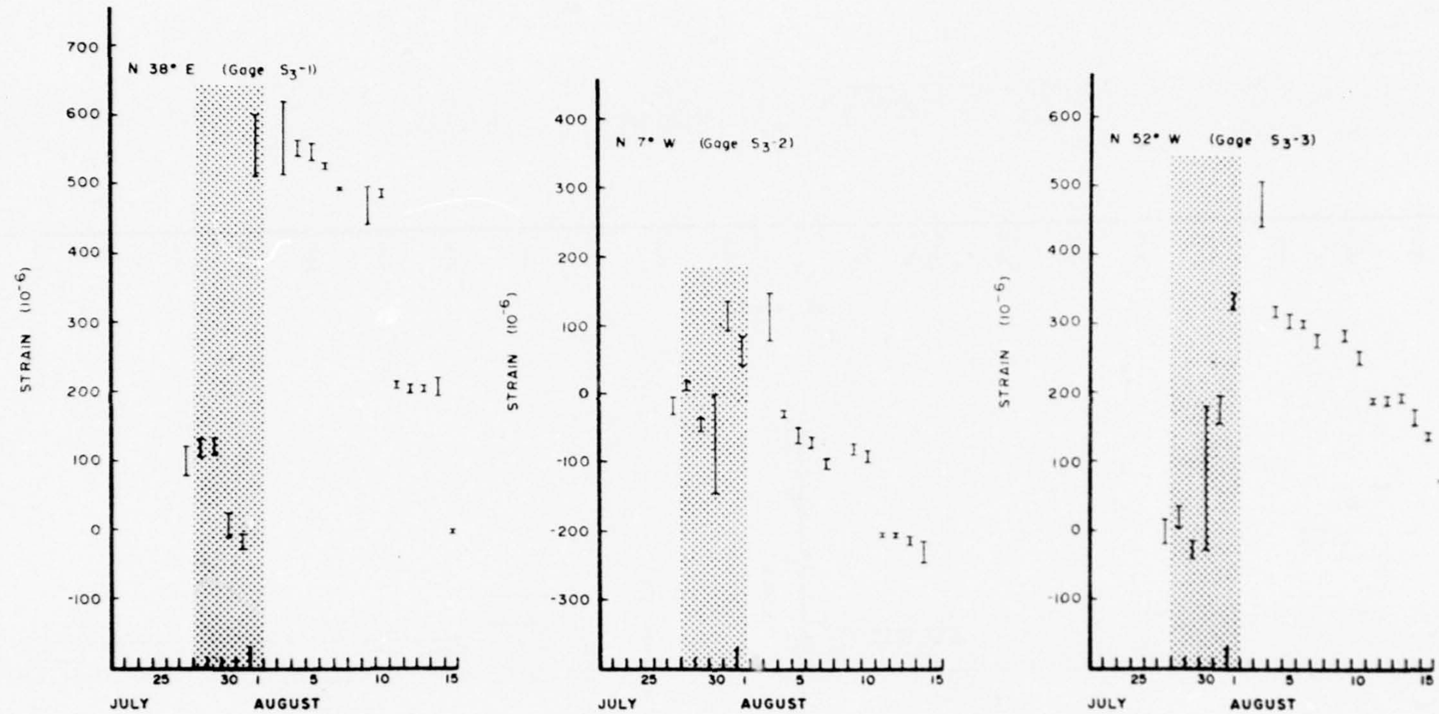

Figure 9. Strain relief monitored by surface strain-gage rosette S_3 during excavation of the test block (shaded region)

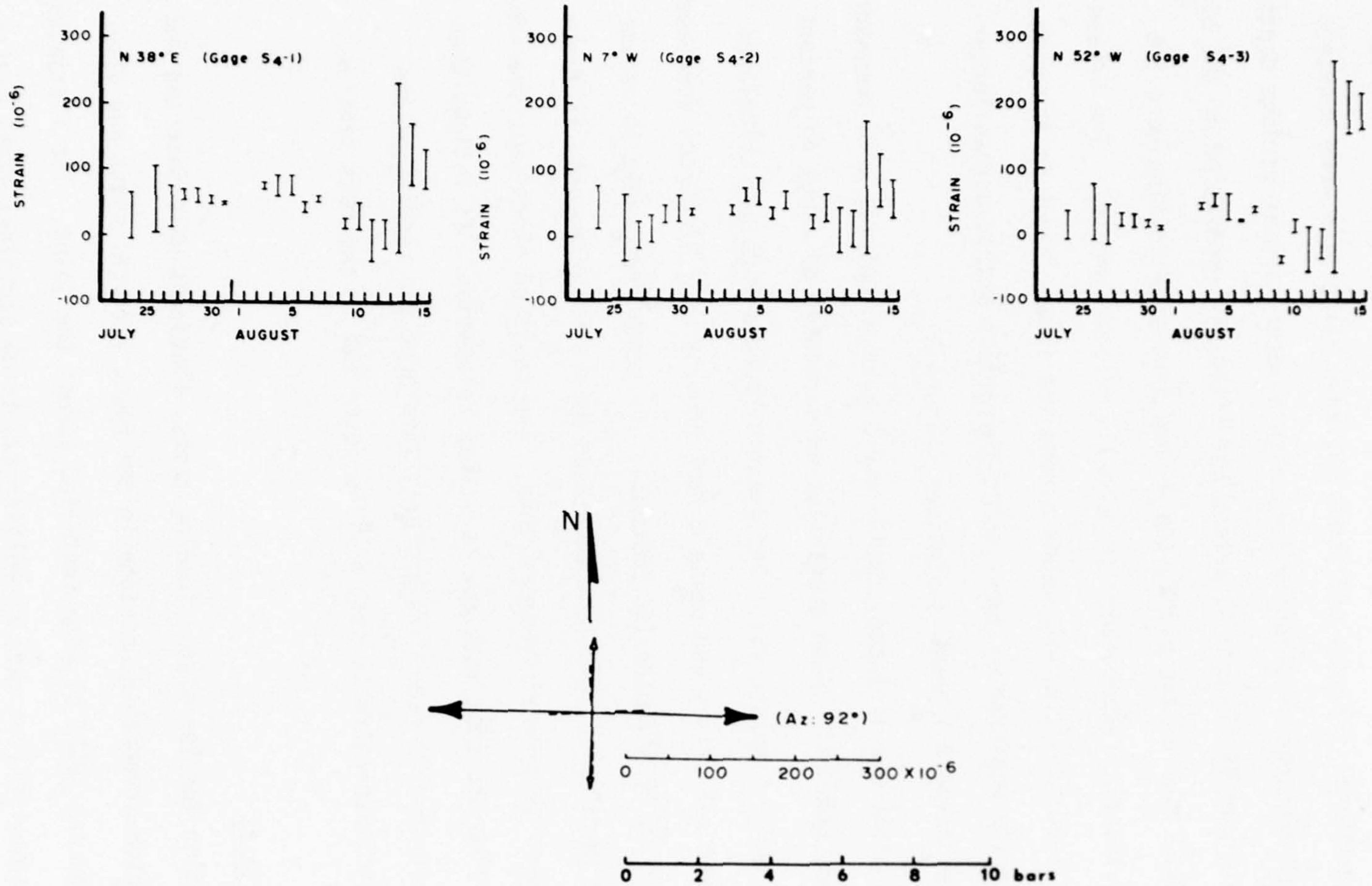

Figure 10. Strain-relief monitored by surface strain-gage rosette S_4 after overcoring on August 13, 1976

measurements across the block. Using shear-wave transducers (70 kilohertz), both the compressional (P) wave and shear (S) wave velocities were measured across a travel distance of slightly less than a meter, and at various depths, from hole VT to hole VR (Figure 4) before and after the excavation of the block. Prior to the isolation of the block, the P- and S-wave velocities were 1.5 km/sec and 0.8 km/sec, respectively (Figure 11, closed symbols). The P-wave velocity determined by ultrasonic means across the intact block is the same as the P-wave velocity measured by the seismic-refraction method across longer travel distances using 0.1 pound explosives (Figure 3).

After the block was isolated, the P- and S-wave velocities were remeasured (Figure 11, open symbols). Both quantities were reduced by about 40 percent to 0.9 km/sec (P) and 0.5 km/sec (S). The dynamic Young's Modulus, calculated from the measured velocities and using a rock density of 1.97 gm/cc, decreased from 33 kilobars to 12 kilobars (60 percent). It seems reasonable to assume that the static modulus of the block decreased by a similar amount, although there is no method to directly measure this. The isolated block may, therefore, be half as stiff as the host rock due to stress relaxation. It follows then that considerable care is needed to properly interpret the results of the subsequent static loading tests that will be described in the next section.

Static Loading Tests

In preparation for the static loading tests, flatjacks were inserted and cemented using high-strength Hydrostone in the open, vertical slots and displacement transducers (DCDT's) were installed across the joint. The flatjacks were slender loading devices made by welding two thin, stainless steel plates (90 x 120 cm) together along the edges. A thin-walled steel tube allowed the entry of water into the flatjacks. Pressurized accumulators were used to in-

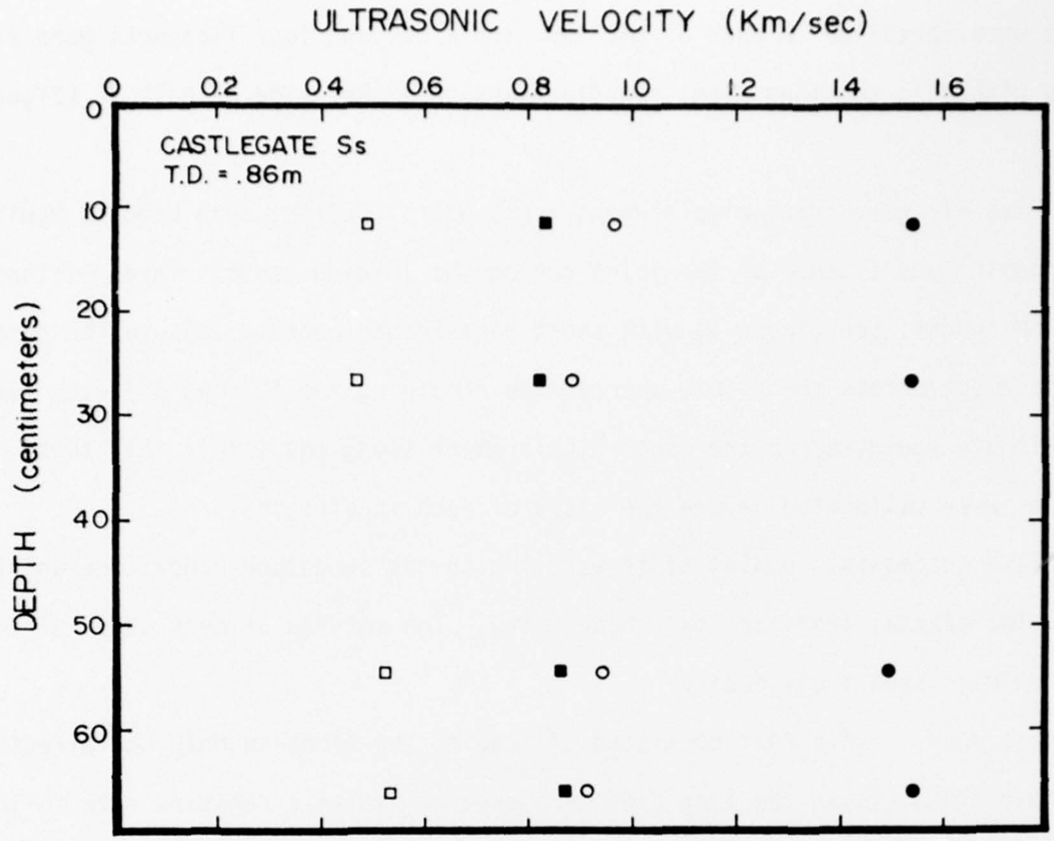

Figure 11. Change in ultrasonic P- and S-wave velocities after excavation of the test block (open symbols). Closed symbols indicate pre-excavation values at various depths.

crease the water pressure inside the flatjacks at a constant rate. Three flatjacks were installed in each of the two side slots and four flatjacks were similarly placed in the long slot, two flatjacks on either side of hole VT (Figure 4).

Five direct-current-displacement-transducers (DCDT's) were used to monitor the opening and closure of the joint during the loading tests. Three of these (SW, SM and SE; see Figure 4) with short gage-length were to measure the normal displacement across the joint, whereas the remaining two (LS and SS) with long gage-length would record the shear displacement along the joint. All these devices were calibrated before the start of each loading test.

Five successful loading tests were run on the sandstone block: one uniaxial test, two biaxial tests and two shear tests. The results of each group of tests will be discussed individually.

Uniaxial Test: This test consisted of loading the block in only one direction. The four flatjacks in the long slot were pressurized at a constant rate to load the block to a peak stress of about 33 bars. The recorded displacements with increasing applied stress are shown in Figure 12. Only two of the normal displacement transducers were functioning during the test (SW and SM; Figures 12A and B) and show increasing closure of the joint during the compressive cycle of the test at a rate of about 3×10^{-3} cm/bar. The displacements parallel to the joint recorded on LS and SS (Figure 12C and D) show what appears to be contrary behavior: the short-shear transducer indicates shortening along the joint, whereas the long-shear (LS) transducer records a slight expansion. The LS transducer monitors almost the entire length of the joint and, therefore, registers the Poisson effect in the block under uniaxial load. The SS transducer monitors a little over half the joint length, but its behavior is not understood.

All displacement curves show a distinct inflection point at about 7 bars

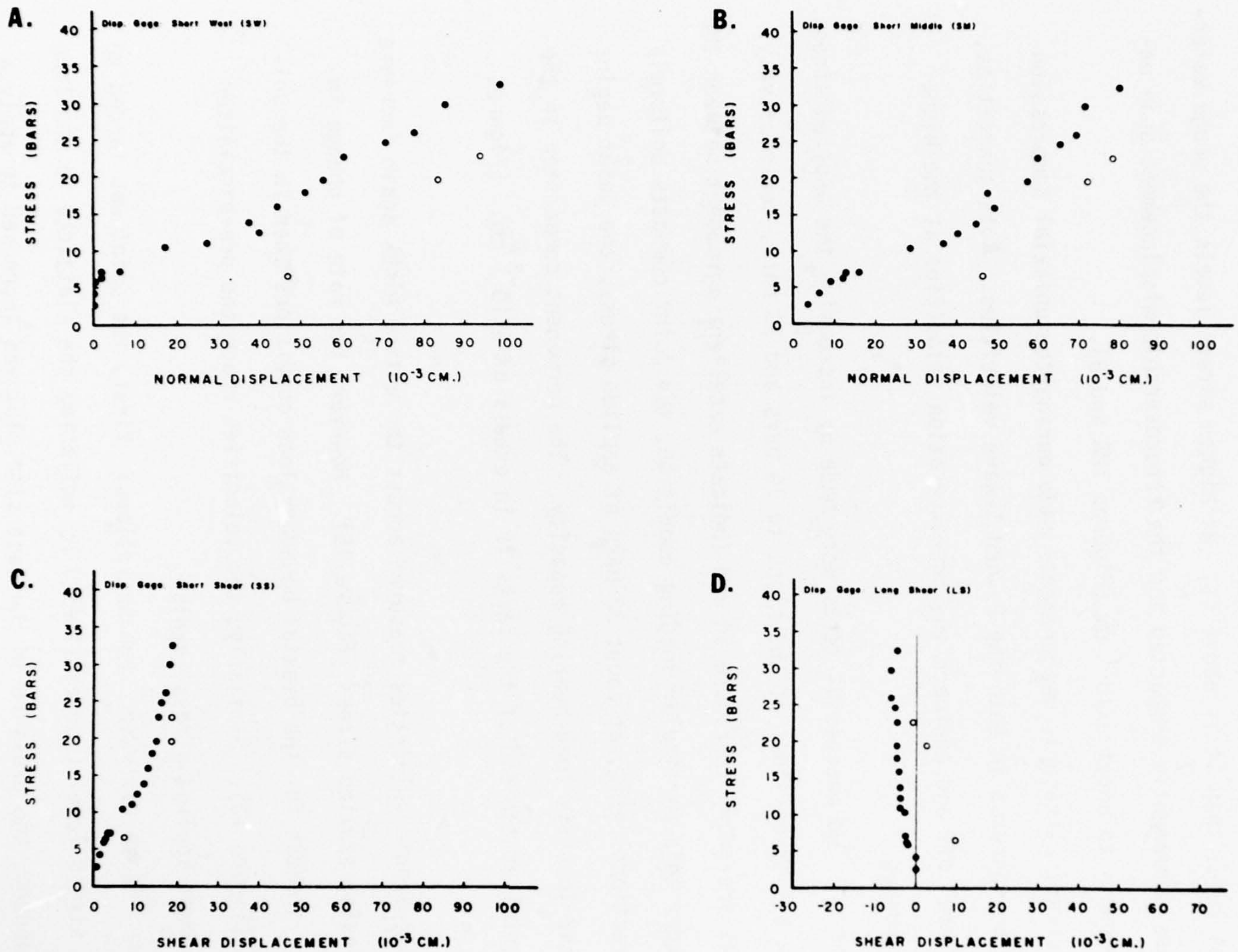

Figure 12. Uniaxial stress - displacement curves for the uniaxial test.
A and B, normal displacement across joint.
C and D, shear displacement along joint.

of applied stress. Recalling that this stress is about equal to the magnitude of the pre-existing stress component normal to the joint, it appears that the behavior of the gouge material in the joint is essentially different below the pre-stress level than it is above it. At higher stress levels the gouge material becomes permanently compacted and the irrecoverable displacement upon unloading amounts to about 3×10^{-2} cm (Figures 12A and B).

Concurrent ultrasonic measurements made during the uniaxial compression test show an increase in both the P- and S-wave velocities. Both quantities, however, level off and approach the pre-excavation velocities at the higher applied stresses.

Biaxial Tests: Two successful tests were made by increasing the applied stress equally on all three sides of the block to 16 bars and 33 bars, consecutively. The results are shown in Figure 14 and indicate excellent agreement between the tests. Under this particular loading condition, the joint compacts uniformly in all directions until, at about 33 bars of applied stress, the joint begins to creep and lose its load-bearing capacity. The permanent compaction in the joint material at the end of the tests is in excess of 6×10^{-2} cm. (Figures 14A and B).

The ultrasonic velocities measured across the entire block again increase with increasing applied stress (Figure 15). However, the rate of change in velocity is greater for the biaxial boundary-load conditions than in the uniaxial case (Figure 13). Similarly, the velocities reach the pre-excavation values at lower applied stress levels.

Shear Tests: These tests were run as follows: first, the joint was loaded to a constant stress level (normal stress) by inflating the flatjacks in the long slot and, second, the pressure in the east side-slot was increased to apply a uniformly increasing shear stress on the joint. The normal stress in the first

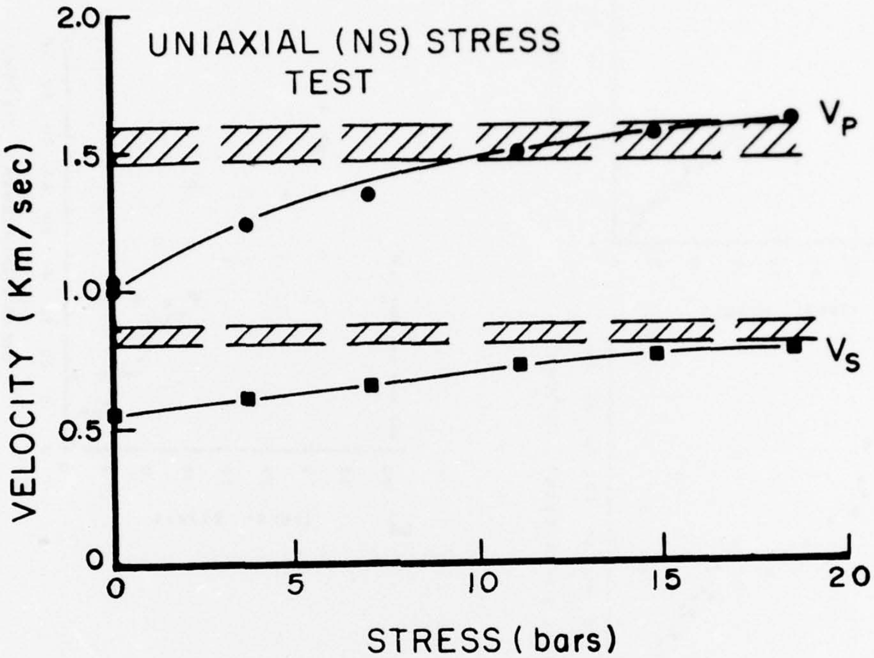

Figure 13. Velocity - stress curves during the uniaxial test. Horizontal broken bars indicate the magnitude of velocities measured prior to block excavation.

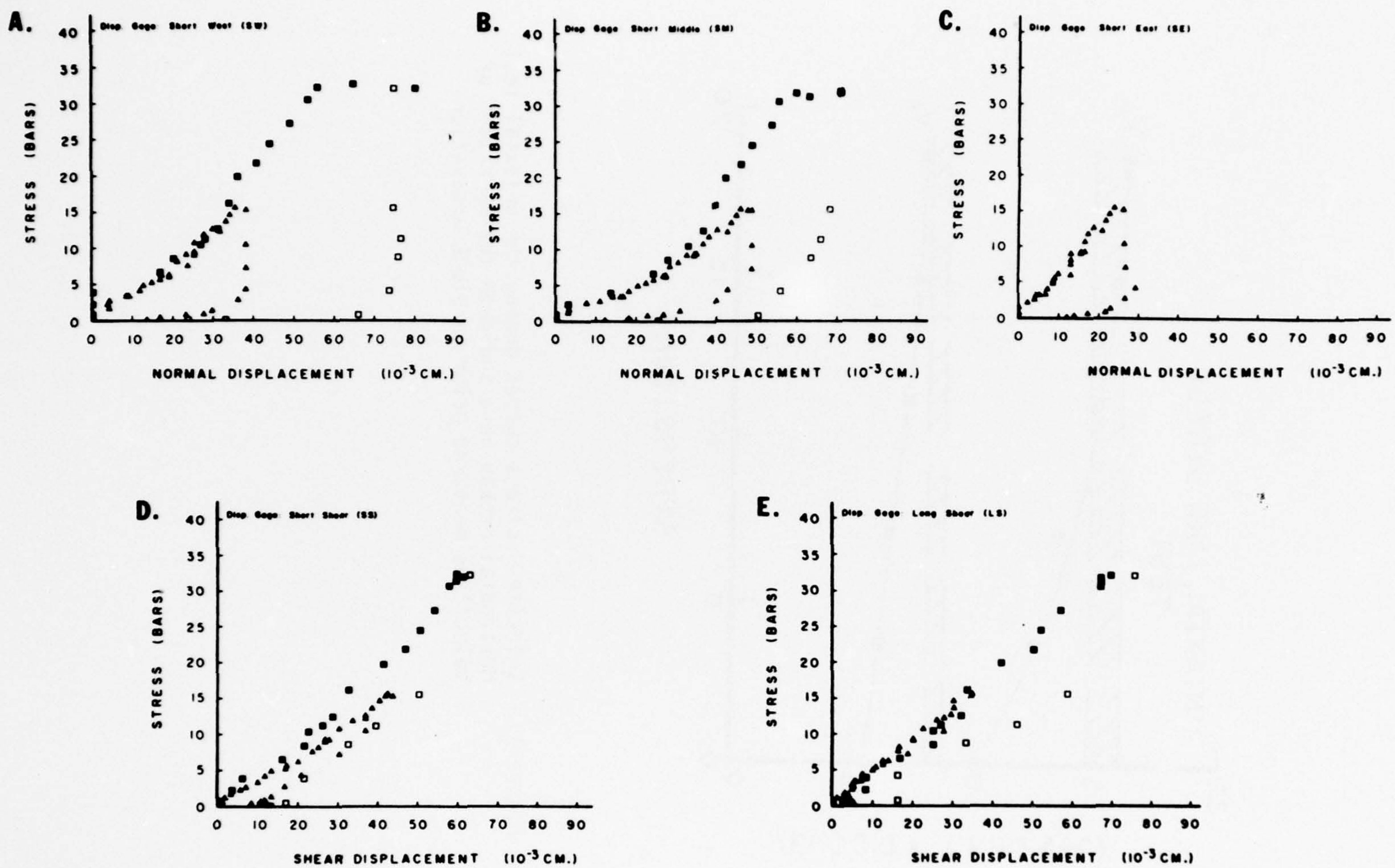

Figure 14. Biaxial stress - displacement curves during biaxial tests (second test - triangles; third test - squares).
A, B and C, normal displacement across joint.
D and E, shear displacement along joint.

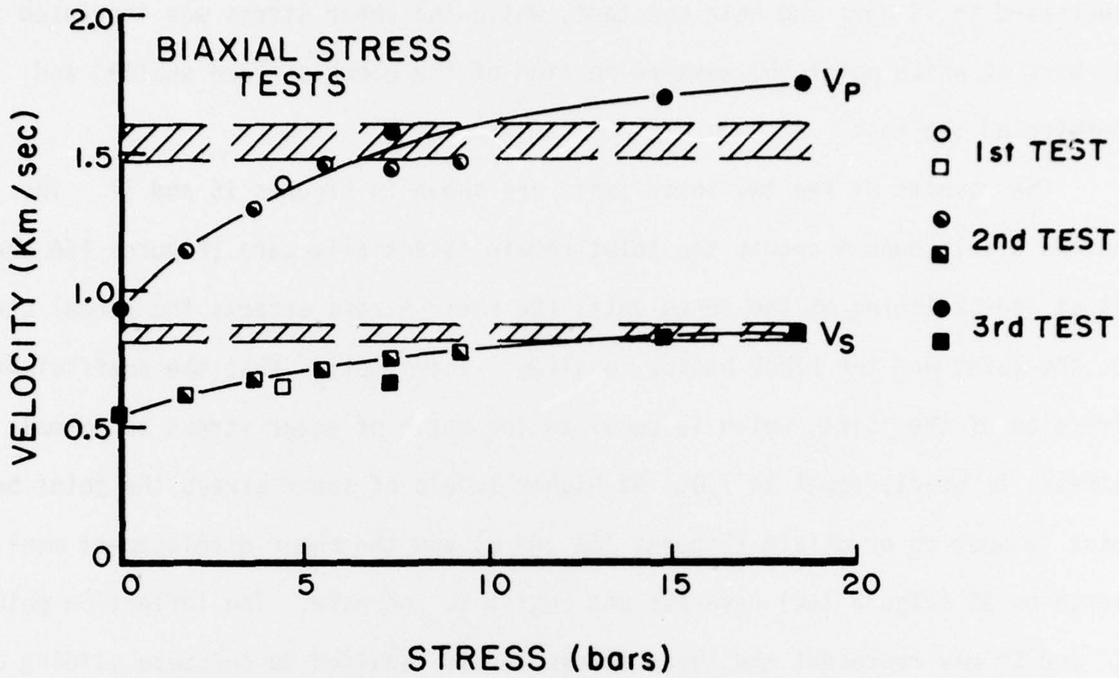

Figure 15. Velocity - stress curves during biaxial tests.

test was held constant at about 7 bars and the shear stress was increased to about 14 bars and then relieved. In the second test, the normal stress was increased to 14 bars and held constant, while the shear stress was increased to 31 bars at which point the eastern portion of the block surface spalled and terminated the test.

The results of the two shear tests are shown in Figures 16 and 17. The normal displacements across the joint remain essentially zero (Figures 16A and B) at the beginning of the tests until the shear stress exceeds the normal stress on the joint and the block begins to slide. This implies that the coefficient of friction of the joint, which is equal to the ratio of shear stress to normal stress, is nearly equal to 1.0. At higher levels of shear stress the joint begins to open up or dilate (Figures 16A and B) and the shear displacement monitored on SS (Figure 16C) reverses and begins to increase. The inflection points C' and C" may represent the shear stress levels required to *initiate* sliding on the joint at two different normal stresses. Figure 17 shows the shortening recorded on LS as the block is compressed during the two shear tests. No obvious change in the rate of displacement, however, is noticeable in the displacement curves.

It should be noted, at this point, that the block as a whole did not move. This is impossible because it is firmly attached to the host rock at the bottom. The data, however, does indicate that the stress conditions for sliding were momentarily reached. The fact that the stress-displacement curves continue to increase argues against actual block movement. Instead, the top layer of the block failed under compression.

The change in ultrasonic velocities with increasing shear stress is, as expected, not very large after their initial increase due to the application of the normal stresses (Figure 18).

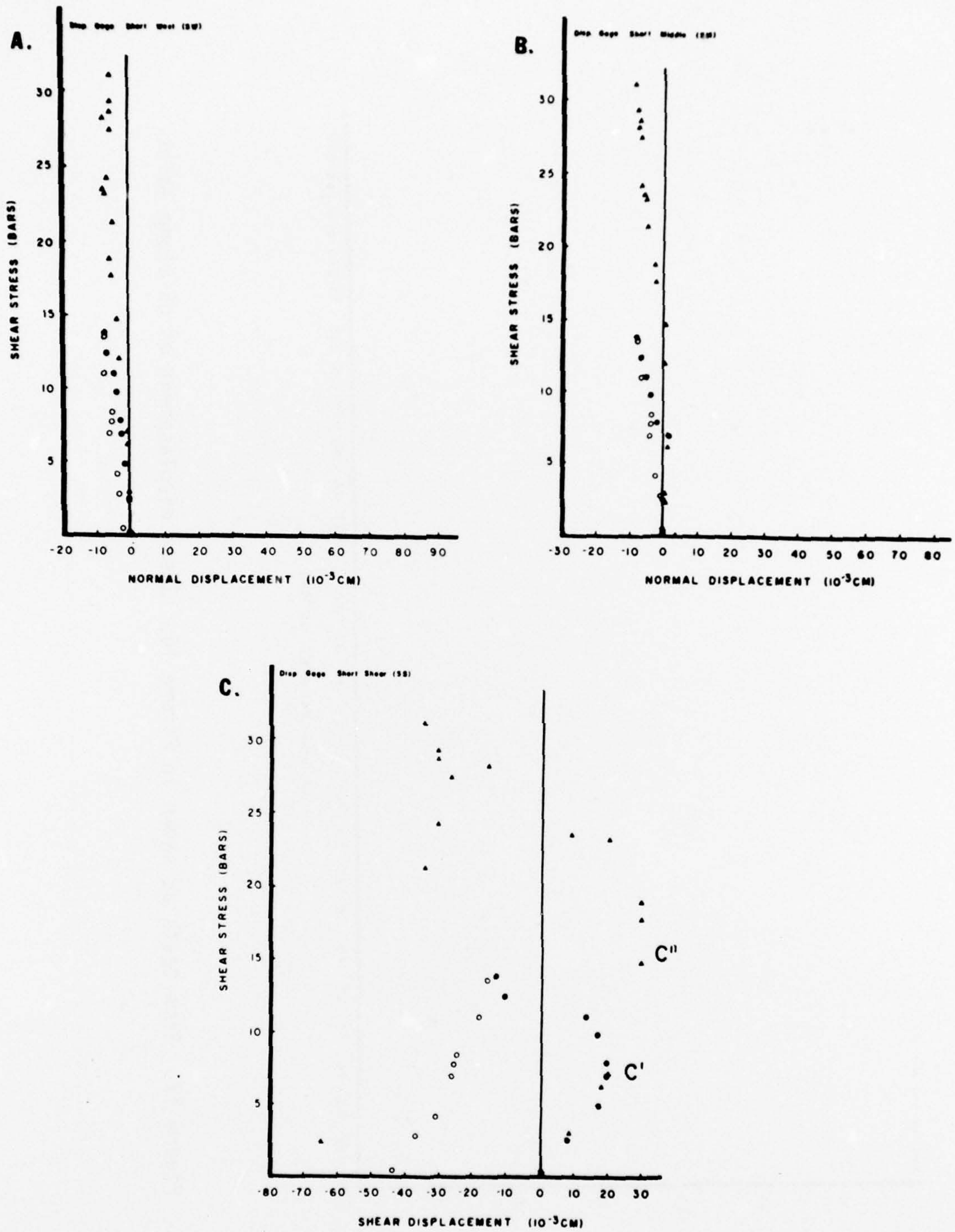

Figure 16. Shear stress - displacement curves during shear tests (first test - circles; second test - triangles). A & B, normal displacement across joint. C, shear displacement on SS along joint. Inflection points C' and C" indicate reversals in displacements and initiation of sliding along the joint.

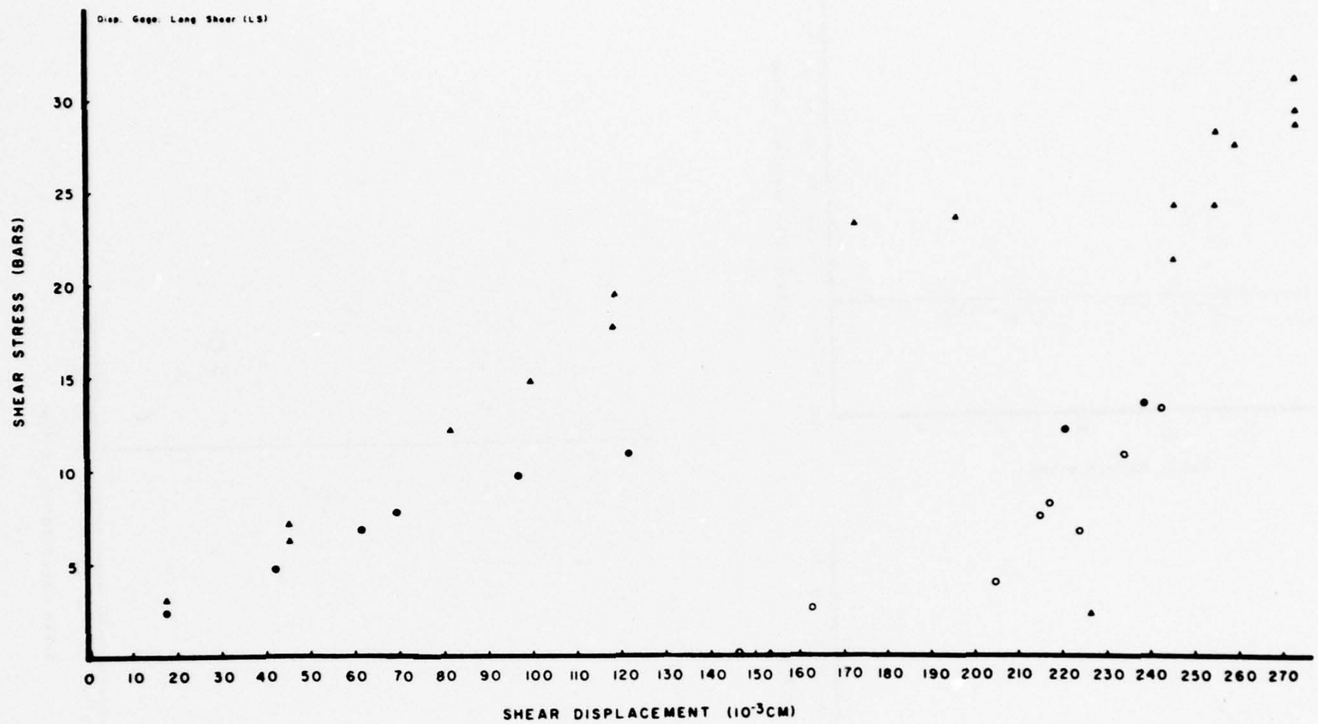

Figure 17. Same tests as shown in Figure 16. Shear displacement on LS along joint.

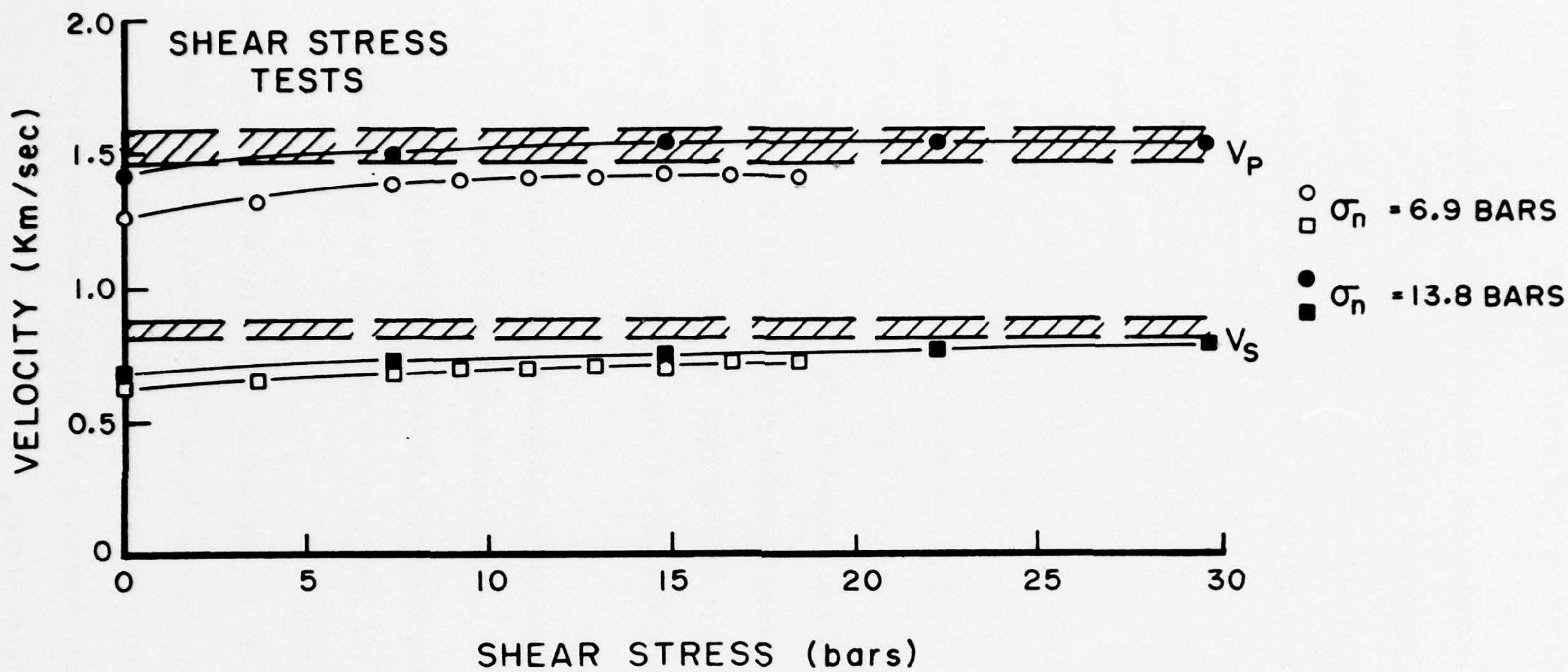

Figure 18. Velocity - stress curves during shear tests for two different normal stresses.

DISCUSSION AND CONCLUDING REMARKS

The combined laboratory and field investigation of a jointed sandstone has yielded the following important observations:

(1) The sandstone mass is pre-stressed to a moderate level of about 10 bars. The maximum horizontal stress component (15 bars) is oriented sub-parallel to the east-west trending joint set. The magnitude of the minimum horizontal stress component perpendicular to the joint set is about 7 bars. These measurements are consistent with earlier results obtained from other localities in the Rangely Anticline (de la Cruz and Raleigh, 1972).

(2) The P- and S-wave velocities in the sandstone depend on both the pre-stress and the moisture content of the rock, but show no dependence on the orientation and frequency of near-vertical joints. By submitting core samples of the sandstone to a humid environment in the laboratory, the velocities are reduced from their "dry" values by about 20 percent and approach those measured in the intact sandstone. This implies that the moisture content in the rock mass is relatively high and that most of the moisture is absorbed at the grain surfaces.

Further reductions in the sonic velocities are the result of the relief of stored strain-energy due to excavation and isolation of the block sample. The attendant deterioration of the block sample is indicated by a significant reduction of the moduli. Of course, it is not known to what degree the strength of the rock has been affected.

The increase in both the P- and S-wave velocities with increasing applied stress is due primarily to the closure of minute cracks, grain boundaries and pores and the attendant increase in the moduli of the rock. The rate of velocity increase, however, depends on the kind of boundary stresses applied; i.e., uniaxial versus biaxial stress.

Significant, however, is the observation that the velocities level off at applied stress magnitudes nearly equal to the pre-stress and that they do not rise much above the "limits" set by the velocities measured in the undisturbed sandstone. Similar results have been obtained in tests on the Kayenta Sandstone near Grand Junction, Colorado (Swolfs and others, 1976). Because none of these block samples were loaded to failure, it is not known whether the velocities will increase again, maintain constant values or decrease slightly until wholescale failure of the sample occurs under continued loading. The fact that large samples tend to fail at lower stresses than small samples of the same material, argues against further increases in velocity. If this is true, then immanent failure of the block sample is indicated once the velocities cease to increase under higher loads.

(3) The dominant, east-west trending joints contain a compacted and indurated gouge material that consists mostly of very fine-grained calcite. The porosity, permeability and grainsize of this material is less than that of the host rock. These joints, although thin enough to be seismically transparent, are impervious to fluids. For this reason, attempts to measure joint permeability under applied stress were singularly unsuccessful.

(4) The "yield" strength of the joint under combined loading conditions is about 33 bars. Above this stress level excessive deformation (creep) occurs. Upon unloading, the permanent compaction in the joint material amounts to about half the joint width.

The shear stress necessary to initiate sliding along the joint (differential movement between block and host rock) depends on the magnitude of the applied normal stress. The coefficient of friction of the joint is approximately equal to one. At higher shear-stress levels the joint begins to dilate, although actual movement of the block does not occur.

The stresses required to cause permanent compaction under normal loading and dilation under shear loading are of the same order of magnitude as the pre-stress in the rock mass. It appears, therefore, that, under *in situ* conditions, differential movement on joints may occur at stress levels that are only slightly higher than the ambient stresses in the rock mass.

Several contrasting comparisons can be drawn between a jointed sandstone and a jointed granite. First, whereas the essentially gouge-free joints in the granite remain the principal conduits for flow of water under applied stresses greater than 20 bars (Pratt and others, 1977), the gouge-filled joints in the sandstone are effective permeability barriers and flow through the host rock predominates. Second, the strength and stiffness of intact granite is greater than that of the joints. On the other hand, no significant differences were observed in the behavior of host rock and joint in the sandstone. Finally, the sandstone mass, even though fractured, appears to be isotropic and homogeneous in the horizontal plane, whereas the jointed granite is highly anisotropic.

The implications of this research on technology are profound in several respects. Despite the fact that laboratory tests can be conducted on rock samples with increasing sophistication, the results when extrapolated to the natural environment are generally inadequate and often misleading. For example, complex calculations performed to predict the ground motion in massive rock sites due to explosive impacts using laboratory data have fallen seriously short of the measurements of actual ground behavior (Johnson and others, 1974). The factors that have contributed to this discrepancy are manifold, but chief among these is the lack of adequate knowledge of site-specific properties. The associated problem of siting strategic facilities

in pre-stressed and fractured rock environments and rendering them safe from explosion-induced or triggered block motion has been reviewed by Rawson (1977).

An important aspect of the research, therefore, is the identification and measurement of those properties that are characteristic of the *site*, as opposed to those typical of the rock itself. Clearly, this can only be accomplished during a properly designed site investigation.

In this report we have summarized the progress made in improving this situation. Even then, considerable improvements in field techniques and methods are needed to quantify the ambient stress and failure criteria associated with fractured rock environments.

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